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Structural behaviour of a collar construction made of frozen soils in a deep excavation

Conduite d'une col de sol congelé dans une excavation profonde

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ABSTRACT

For the renewal and extension of the Central Metro Station in Rotterdam, the construction of a retaining wall made of frozen soil as a collar around the existing tunnel turned out to be the optimal solution. The water tightness and structural stability of the frozen soil wall and its connections to the diaphragm walls of the building pit was strongly required. This paper describes the results of laboratory tests on frozen soil samples and the results of 2D FEM calculations to determine the forces in the frozen soil wall. The effects of creep are taken into account for the determination of the long term effects.

RÉSUMÉ

Pour la rénovation et l'extension de la Station de Métro Centrale de Rotterdam, la solution constructive optimale était l'application d'une paroi de sol congelé. La perméabilité et la stabilité de la paroi de sol congelé et les jonctions avec les rideaux de parois moulés en béton armé étaient des points d'importance du projet. Dans cette contribution, les résultats des essais de laboratoires et des calculs appliquant la méthode des éléments finis en 2 dimensions pour déterminer les forces dans la paroi de sol gelé sont traités. Les effets de fluage sont pris en compte pour la détermination des effets de longue durée sur la qualité des matériaux congelés.

Keywords : frozen soil, deep excavation, underground structures, retaining wall, FEM

1 INTRODUCTION

In Rotterdam the connection will take place of the existing metro station to RandstadRail, a future light rail connection between Rotterdam, The Hague and Zoetermeer. This metro station is an important junction for public transport in Rotterdam. For this purpose the existing underground metro terminus has to be enlarged. Meanwhile the metro traffic has to be continued without any disturbance. The building pit, needed for the new station, was executed with diaphragm walls. At the east side of the building pit an almost semicircular collar construction had to be realized around the existing metro tunnel using the soil freezing technique. The design of the collar is complicated because of its shape and the use of frozen soil as a retaining wall. In this paper the construction of the underground station and the collar construction are described. The determination of the frozen soil parameters will be described and the results of the Finite Element design of the collar construction will be discussed.

2 GEOTECHNICAL CONDITIONS

The geotechnical profile of the station area shows below surface level (at Dutch datum level NAP) an anthropogenic sand layer with a thickness of 3 to 5 m, underneath which Holocene clay and peat layers with a thickness of about 10 m are present, followed by a Pleistocene sand layer with a thickness of about 20 m. At NAP –15 m locally a sand lens with a thickness of 1 m is found. The Pleistocene sand layer lies on top of the so called Formatie van Kedichem, consisting of low permeable sandy loam, peat and clay layers. The freatic level lies at about NAP –2,0 m. The piezometric head in the highly permeable Pleistocene sand lies at about NAP –2,0 m. Figure 1 shows a geotechnical profile of the area.

3 EXISTING STATION

The existing terminus has a width of about 13 m and a height of about 6 m. The bottom level of the tunnel lies at NAP –10 m. It was built in 1960 as a submerged tunnel. It is composed of sections each with a length of 40 to 50 m. The tunnel is founded on concrete piles with a diameter of 0,5 m which were applied before the submerging of the tunnel. The connection between the tunnel and the piles is able to transfer vertical compression force and shear force. The space below the tunnel floor (about 1 m in thickness) was filled with sand directly after submerging.

4 NEW STATION

The new station is being built at the location of the existing terminus. While the metro traffic has to be continued undisturbed, the foundation of the existing tunnel has to be adapted and reinforced. The walls of the building pit are formed by diaphragm walls, which reach to a level of NAP –38 m., into the impermeable Kedichem layers. Using this method the excavation of the building pit can take place without lowering of the piezometric head. This is an important factor in the possibility to create a deep excavation in the city of Rotterdam. The ultimate excavation level is NAP –15 m, so it reaches also below the existing tunnel sections.

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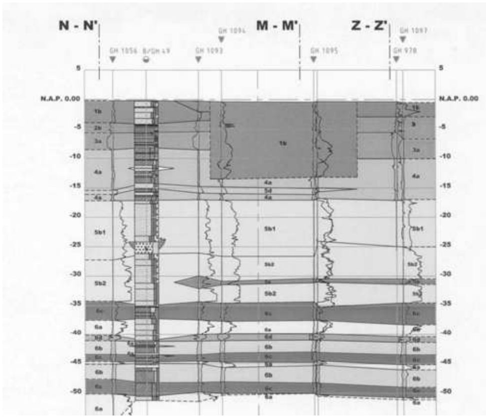

Figure 1. Geotechnical profile.

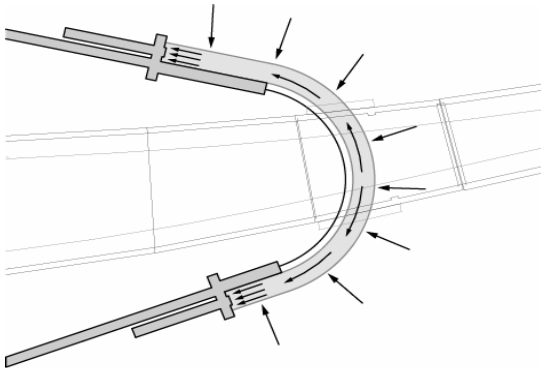

Figure 2. Top view frozen soil body.

4.1 *Collar construction*

At the east side the building pit had to be closed around the existing tunnel with a collar construction. This closure has to possess sufficient strength to resist the soil- and water pressures. Besides it has to be watertight as leakage would cause a lowering of the freatic level outside the building pit, which has to be prevented. The retaining height of the collar construction is 15 m. of which 5 m reach underneath the tunnel. The wall has to be applied to a level of NAP –38 m, into the impermeable Formatie van Kedichem, to prevent the water in the Pleistocene sand to penetrate into the building pit. The collar is designed as a semicircular wall with a diameter of about 30 m.

4.2 *Construction of the collar construction*

The semi circular shape of the frozen soil is chosen to reduce the bending stresses caused by the soil and water pressures at the outside. In this type of construction also tangential stresses occur. The horizontal deformation of a semicircular wall are much lower compared to those of a flat wall with the same span, depending on the diameter of the circle. The tangential forces have to be resisted by the connecting diaphragm walls, as shown in Figure 2. From a preliminary analytical calculation and adopted strength of the frozen soil, the minimal thickness of the wall was determined at 2,5 m. The soil freezing method was chosen in this situation because of it's possibility to control the dimensions, strength and watertightness of the wall at a low level of risk compared to methods like the jetgrouting technique.

The wall was formed in frozen soil, both aside and underneath the tunnel by applying vertical freezing pipes alongside and through the tunnel to a depth of NAP –38 m. The freezing process was executed with liquid nitrogen (at a temperature of – 196 ºC) in the so called freeze-up stage and continued with brine (at a temperature of –37 ºC). The freezing rate is depending on the soil type and the permeability of the soil outside the frozen soil body. Clay and peat layers freeze at a much lower speed than sand layers. The excavation of the building pit was executed in stages of about 2,5 m. After each stage an in situ concrete wall with a thickness of 0,85 m was applied against the frozen soil wall. The freezing process had to be maintained until the in situ concrete walls and the floor of the of the new station were completed, which was a period of at least 650 days according to the contractor's planning. The freezing period ended in February 2009. An overview of the project is given in Thumann et al. (2009).

5 FROZEN SOIL

5.1 *General*

During the freeze-up stage the temperature of the soil around the freezing pipes decreases very rapidly. The rate of the growth of the ice layer is depending on the temperature gradient and the thermal conductivity of the soil. The temperature gradient is determined by the distance between the freezing pipes. The thermal conductivity differs in each soil layer. This means that the frozen soil body will reach a thickness which is varying over the depth. The thickness will be the largest in sand layers and the relatively small in clay and peat layers. When the required minimum thickness was reached the supply of cooling liquid was limited to maintain a more or less constant thickness of the wall.

5.2 *laboratory investigations*

The strength and stiffness parameters of the frozen soil were determined in a laboratory investigation. The tests were performed in cooperation with CDM Jessberger in Bochum (Germany). The tests were performed on samples at a temperature of -10ºC and –20 ºC. on all relevant soil types, sampled from the construction site. In Table 1 some properties of the unfrozen soil are given as determined from laboratory tests as mean values in the area of the centre of Rotterdam.

Table 1. Properties unfrozen soil.

Soil type	Volumetric weight (kN/m^3)	φ' (˚)	c' (kN/m^2)	$E_{50;ref}$ (kN/m^2)
Sand fill (1b)	19,5	36,2	0	27600
Silty clay (2b)	16,5	25	5	7500
Peat (3a)	10,1	12,4	20	3700
Silty clay (4a)	16,4	16,6	18	6650
Pleistocene sand (5b)	20,0	30,0	0	20000
Holocene sand (5d)	19	27,5	0	7500

5.2.1 *Uniaxial tests*

In Table 2 the results of the compressive strength and Young's modulus at a temperature of –10 ˚C and –20 ˚C (between brackets) are given as a mean value of the tests on the different soil layers.

Table 2. Results from Uniaxial compression tests.

Soil type	Number of tests	Compressive Strength (MN/m^2)	Tangent Modulus (MN/m^2)
1b	4 (3)	18,9 (20,9)	1163 (1251)
2b	1 (1)	3,4 (8,5)	338 (271)
3a	3 (3)	3,0 (8,9)	410 (1833)
4a	3 (2)	3,7 (7,6)	426 (779)
5b	4 (3)	11,3 (17,1)	1021 (2703)
5d	4 (3)	7,7 (11,1)	1196 (1266)

From this table it can be concluded that the compressive uniaxial strength is depending on the temperature. In most cases also the Young's modulus is increasing at lower temperatures, apart from the results in the silty clay(2b). The tensile strength of the frozen soil in the was not investigated in the laboratory. In the structural calculations (see 6.1) the tensile strength is taken as:

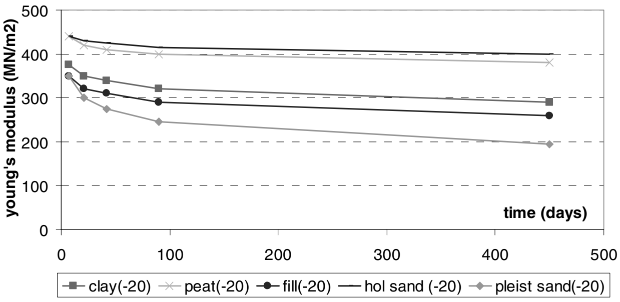

Figure 3. Relation between Young's modulus and time

$$\sigma_t = 0{,}2\,\sigma_u \qquad (1)$$

where σ_t = ultimate tensile strength and σ_u = ultimate compressive strength.

5.3 *Creep*

Creep of frozen soil differs from the creep of soil at temperatures above freezing point. The creep of frozen soil is strongly depending on the stress level in the soil. Though creep also varies in different soil types, the effect occurs in every soil, both in clay, peat and sand layers. The change in the mechanical properties of the frozen soil in time differs very much. The creep effect is taken into account with:

$$\varepsilon = \frac{\sigma_1}{E_0} + A\,\sigma_1^B\, t^c \qquad (2)$$

where A,B,C are creep test parameters, E_0 = initial Young's modulus, σ_1 = constant axial stress and t = time.
Creep tests were performed on the diferent soiltypes. The results were used to determine the long term strength and stiffness of the frozen soil.

5.3.1 *Triaxial tests*

The triaxial tests taken from a boring at the specific area of the building pit were performed at room temperature (+10 ˚C) and at –10 ˚C. The results are given in Table 3.

Table 3. Results from triaxial tests.

Soil type	Test at +10 ˚C		Test at –10 ˚C	
	φ (˚)	c MN/m^2)	φ (˚)	c (MN/m^2)
1b	-	-	33	3,8
2b	-	-	-	-
3a	-	-	20	1,2
4a	27,2	0,011	5	1,6
5b	32	0	32	2,7
5d	29	0	29	2,3

In Figure 3 the relation between Young's modulus and time due to the effect of creep is given for a temperature of -20˚C for 5 different soil layers. In the design stage of the project the lifetime of the frozen soil construction was supposed to be about 400 days. The time effect is most obvious in the first 100 days. During the total period the stiffness decreases about 40 % in the fill and about 32 % in the clay and peat. Due to the time effect also the cohesion and the compressive strength will decrease in time, as is shown in Figures 4 and 5.

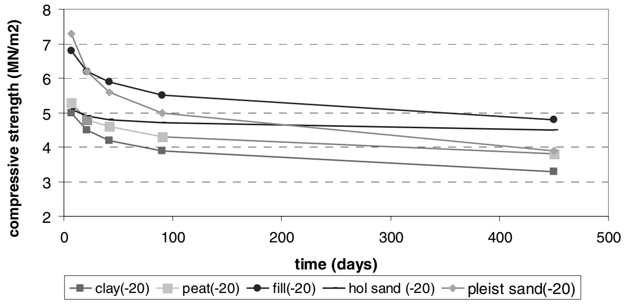

Figure 4. Relation between cohesion and time

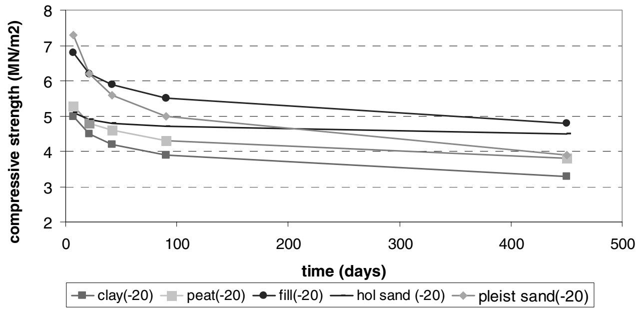

Figure 5. Relation between compressive strength and time

In the clay layers the freezing of the soil causes a strain perpendicular to the axis of the freezing pipes. This so called frost heave effect was also investigated in laboratory tests. The results are outside the scope of this paper.

6 CALCULATIONS

In the design the frozen soil wall was considered as a semicircular construction with a radius of 15,5 m. The stresses and deformations were calculated in a 2D FEM axial symmetric calculation, in which only the part of the wall below the tunnel, from NAP –10 m to NAP –38 m was taken into account.
In the calculations the effect of the excavations was determined. Also the effect of the horizontal frost heave effect was regarded, but is not reported in this paper.

6.1 *F.E.M. calculations*

The aim of the calculations was to determine the bending moments and the axial and tangential forces in the wall. The wall was considered to be connected to the tunnel floor (at NAP –10 m) by means of a sliding plane, which was able to transfer only an axial force from the wall to the tunnel. The Mohr Coulomb Model was applied for the frozen soil, the untreated soil was modeled with the Hardening Soil Model in PLAXIS 2D, version 8.1, see Brinkgreve (1998). Calculations were made both for long term and short term conditions.

6.1.1 *Bending moments*

Figure 6 shows the calculated bending moments in the wall in the excavation stages of NAP -12 m, NAP -14 m and NAP -15 m. The maximum calculated value was –180 kNm/m, occurring with short term parameters of the frozen soil. The bending stresses in the wall remain far below the ultimate strength values as determined from the laboratory tests.

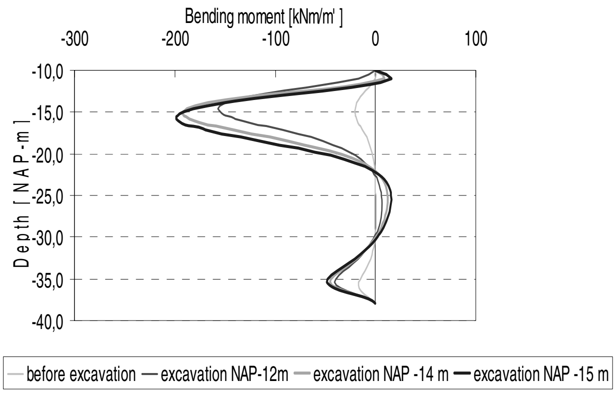

Figure 6. Bending moment during excavation stages.

6.1.2 *Axial force*

During the progress of the excavation the normal force in the wall increases due to the effects of unloading of the soil. This unloading causes a shaft friction along the outside of the wall, which is transformed to the tunnel floor. In Figure 7 the build up of the axial force in the wall is shown at the excavation levels at NAP –12m through NAP –15 m. The results refer to the short time behaviour.

6.1.3 *Tangential force*

The tangential force in the wall is also depending on the excavation level, see Figure 8. The tangential force is also depending on the stiffness of the soil layer. At a level between NAP –15 m and NAP –16 m a stiff holocene sand layer (type 5d, see Table 1) is found. In this layer, lying around the excavation level, the tangential forces reach the highest values of about 1000 kN/m'.

The tangential forces in the wall are transferred to the connecting diaphragm walls. In a 3D FEM calculation the in plane load on the diaphragm wall was calculated, which is not reported in this paper.

6.1.4 *Deformations*

The calculated deformations of the frozen soil wall were relatively small, about 25 mm. This is of importance for the analysis of the behaviour of the tunnel. Measurements during the execution of the work confirm the small movements. The excavation was completed without any disturbance of the metro traffic.

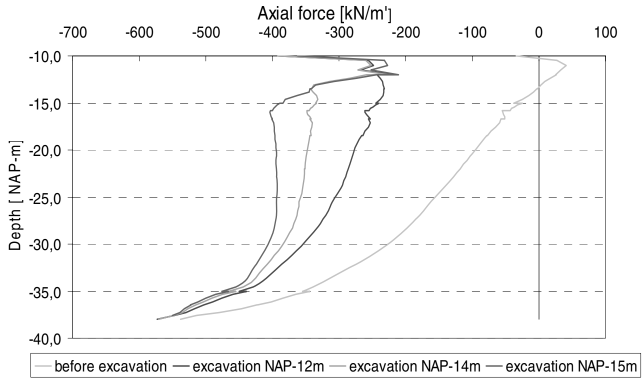

Figure 7. Axial force in the wall during excavation stages.

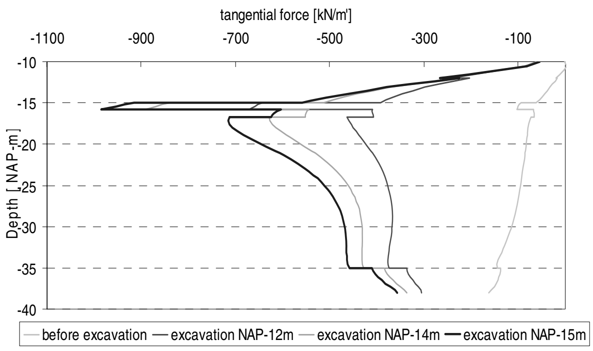

Figure 8. Tangential forces in the wall during excavation stages.

7 CONCLUSIONS

From the soil investigations and calculations the following conclusions can be drawn.

The strength and stiffness parameters of frozen soil appear to be time dependent. All these parameters decrease with time.

The combination of a semicircular wall and the application of the soil freezing technique allows to construct the complex ending of the building pit in a safe way.

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